VOLUME 80 SEPARATE No. 456

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

JUNE, 1954

SAND DRAIN APPLICATIONS BY THE PORT OF NEW YORK AUTHORITY

by John M. Kyle, M. ASCE, and
Martin S. Kapp, J.M. ASCE

Presented at
Atlantic City Convention
June 14-19, 1954

SOIL MECHANICS AND FOUNDATIONS
DIVISION

{Discussion open until October 1, 1954}

Copyright 1954 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

THIS PAPER

--represents an effort by the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final date on which a discussion should reach the Manager of Technical Publications appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Papers" (Proceedings — Separate No. 290). For free copies of this Separate—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference (or paper number), and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

SAND DRAIN APPLICATIONS BY THE PORT OF NEW YORK AUTHORITY

John M. Kyle[1], M. ASCE
and
Martin S. Kapp[2], J.M. ASCE

INTRODUCTION

The use and misuse of sand drains has been the subject of numerous papers and discussions for the past ten years. Opinions range from those calling them a panacea for all settlement and weak soil problems to those who insist the drains defeat their own purpose.

We in the Port Authority have been fortunate in having had presented to us for study, analysis, and finally for construction, a great number of projects for which the sand drain technique appeared to be particularly well suited (see Figure 1 for location of projects). We also had associated with us on one or more of these projects some of the foremost consulting engineers in this field.

You will note in the discussions that follow that our applications have not been completely without mishap. As a matter of principle, where safety was not involved, we have in certain cases carried on investigations by experimenting with increases in the weight and rate of application for surcharge loads. These experiments resulted in developing apparent areas of instability on a portion of the sand drained areas. Because of this procedure, small areas of local failure were induced on two of our major projects.

The Port Authority is fortunate in being both operator and designer of each of the projects discussed so that a calculated risk could be taken during construction in an effort to achieve maximum economy. Most consulting engineers are not so fortunate and must insure complete stability of all portions of the project even during the construction period. This of necessity, in many instances, results in over design.

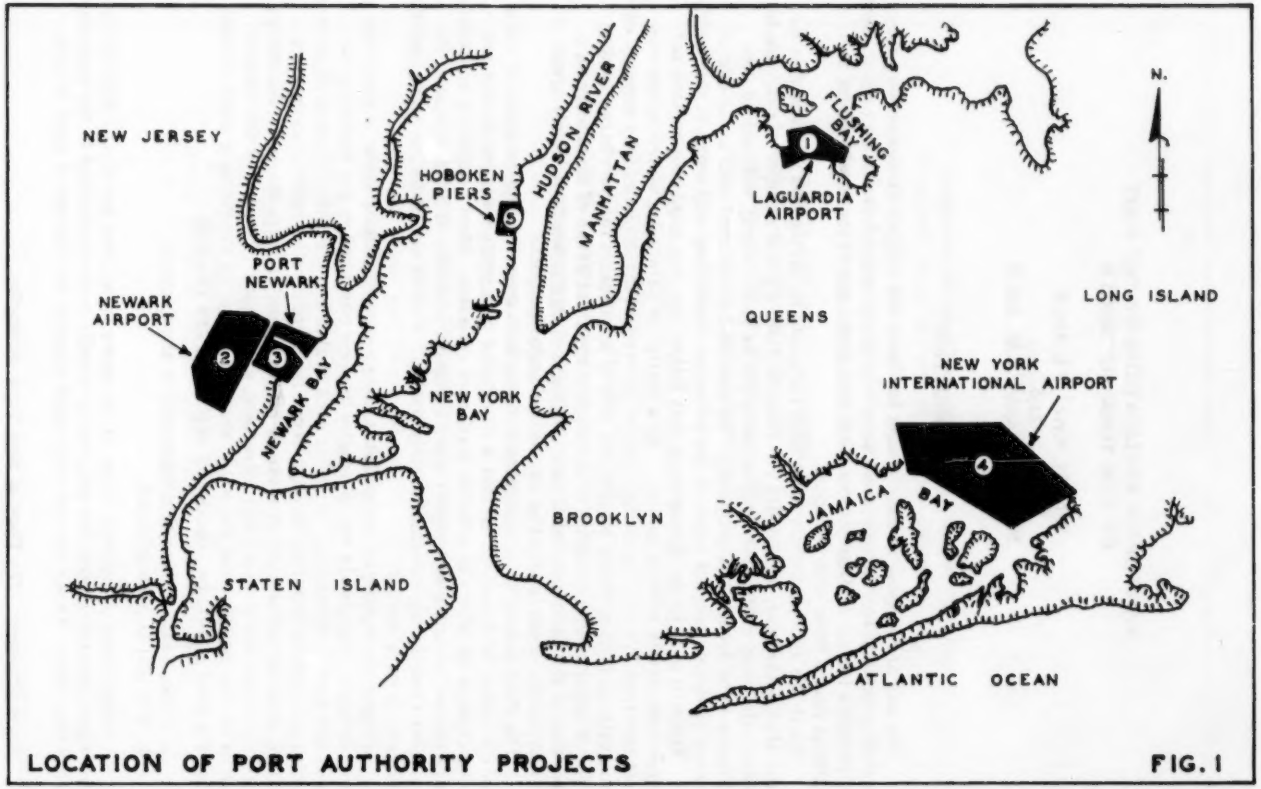
Without attempting to review what most readers already know, a reminder of certain fundamentals which usually become obscured in any technical discussion seems advisable. Vertical sand drains are circular columns of porous sand and gravel installed through deposits of relatively impervious compressible soils to quicken the process of removing water from the soil. Surcharge loads, composed of heavy fill materials, are usually placed over the treated area to help squeeze out the water more rapidly. By providing a quick escape for the pore water, two important objectives are reached.

1. Consolidation (rate of settlement) is accelerated.
2. The soil is strengthened.

Although both objectives occur at the same time, they are listed separately because, depending upon the job, one is usually more important to the engineer than the other. These are the only valid reasons for the use of sand drains.

---

1. Chief Engineer, The Port of New York Authority.
2. Soils Engineer, The Port of New York Authority.



LOCATION OF PORT AUTHORITY PROJECTS FIG. 1

Before evaluating the worth of sand drains, it is necessary to make the following assumption: That part of the theory of consolidation which states that settlement under load is due to a process involving a decrease in water content of a saturated soil without replacement of the water by air holds true. This theory is borne out by the decrease in water content shown for those borings which were taken after settlement had occurred.

There are five separate Port Authority projects that have been completed or are presently underway for which sand drains were considered as an aid to construction. On two of them, sand drains were used for the entire project; on two of them, sand drains were used for a major portion only, and one one of them sand drains were considered but not used at all. Each project is presented in chronological order including both laboratory and field data.

LA GUARDIA AIRPORT
(Project 1)

Location and Geology of Site

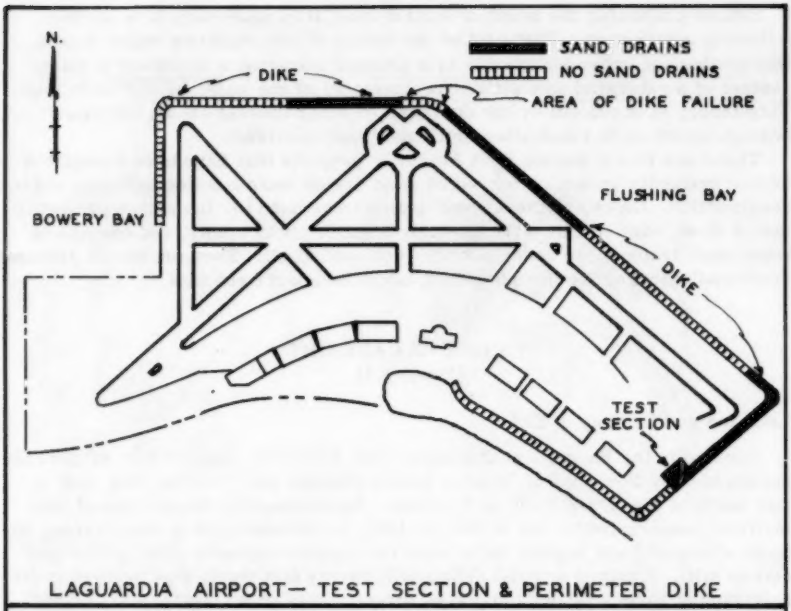
Located in the Borough of Queens of New York City, La Guardia Airport is surrounded by Bowery Bay, Rikers Island Channel and Flushing Bay, and is just north of the Grand Central Parkway. Approximately 70 per cent of this airfield, constructed by the W.P.A. in 1938, is situated over a deep marine deposit of organic silt (a local term used for organic deposits of silty clay and clayey silt). A mixed ash and debris fill, twenty feet thick, was brought in from neighboring Rikers Island and placed directly over this original soft, highly compressible soil to bring the field to grade elevation. Difficulty was encountered during construction in trying to prevent serious mud waves during fill operations, in fact mud waves did occur which considerably slowed down the placement of fill. The original engineering reports on the project indicated that subsequent settlements would take place and additional fill would have to be placed in the future. World War II postponed any contemplated rehabilitation and with the heavy increase in air transportation immediately after the end of hostilities, closing down of the airport or limiting field operations for large scale repairs was deemed impractical. Geologic profile is shown on Figure 2.

The Problem

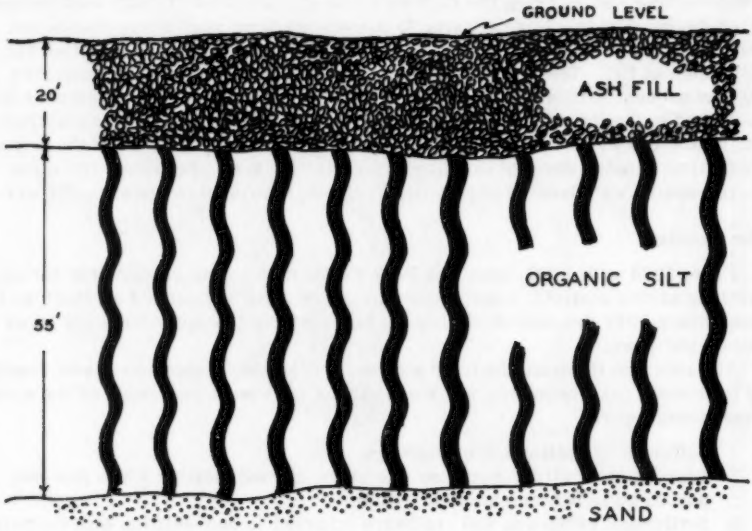
From 1938 until 1947, when the Port Authority became responsible for operations at this airfield, a major portion of the field had settled as much as five feet. Since 1947 the rate of settlement has continued at approximately three inches per year.

Almost from the time the field was opened the city management was beset by problems, each becoming more serious as time went on. Some of the most troublesome were:

1. Differential settlement of runways.
2. Disruption of utility services due to the ground settling while pile supported buildings remained at grade.
3. Settlement of aprons and roadways adjacent to non-settling pile supported structures.
4. Differential settlement of terminal building supported on piles not driven to good bearing.



LAGUARDIA AIRPORT— TEST SECTION & PERIMETER DIKE



SOIL PROFILE — FILLED-IN SECTION OF AIRPORT

FIG. 2

In spite of all this, the field has been able to operate on a full scale basis but it has required constant and costly maintenance.

Starting in 1945, one problem became critical. A combination of strong winds and high tides in the Fall of the year caused flooding of the airport and a resulting halt in plane movements. Since no commercial airport can operate on an intermittent basis, expedient action was required to save La Guardia from becoming a submerged airfield.

An engineering report, submitted to the City of New York in 1946, recommended that sand drains and surcharge fill be employed on runway, taxiway and apron areas. The objective would be to bring all paved areas to a higher elevation and minimize future settlement. This program would have required closing the airfield for at least one year. Because of the prohibitive cost and the necessity for having the field in operation at all times, this plan was rejected.

Intermediate Test Program

Immediately after assuming management of the airport in 1947, The Port of New York Authority instituted a research program on possible methods of stabilizing the airport. A group of foundation engineering consultants supervised this program which consisted of:

1. Complete soils investigation.
2. Installation of sand drained test sections to check the effectiveness of sand drain techniques.
3. Investigation of methods of stabilization other than sand drains.
4. A report and recommendations on protection of the airfield to keep it in operation.

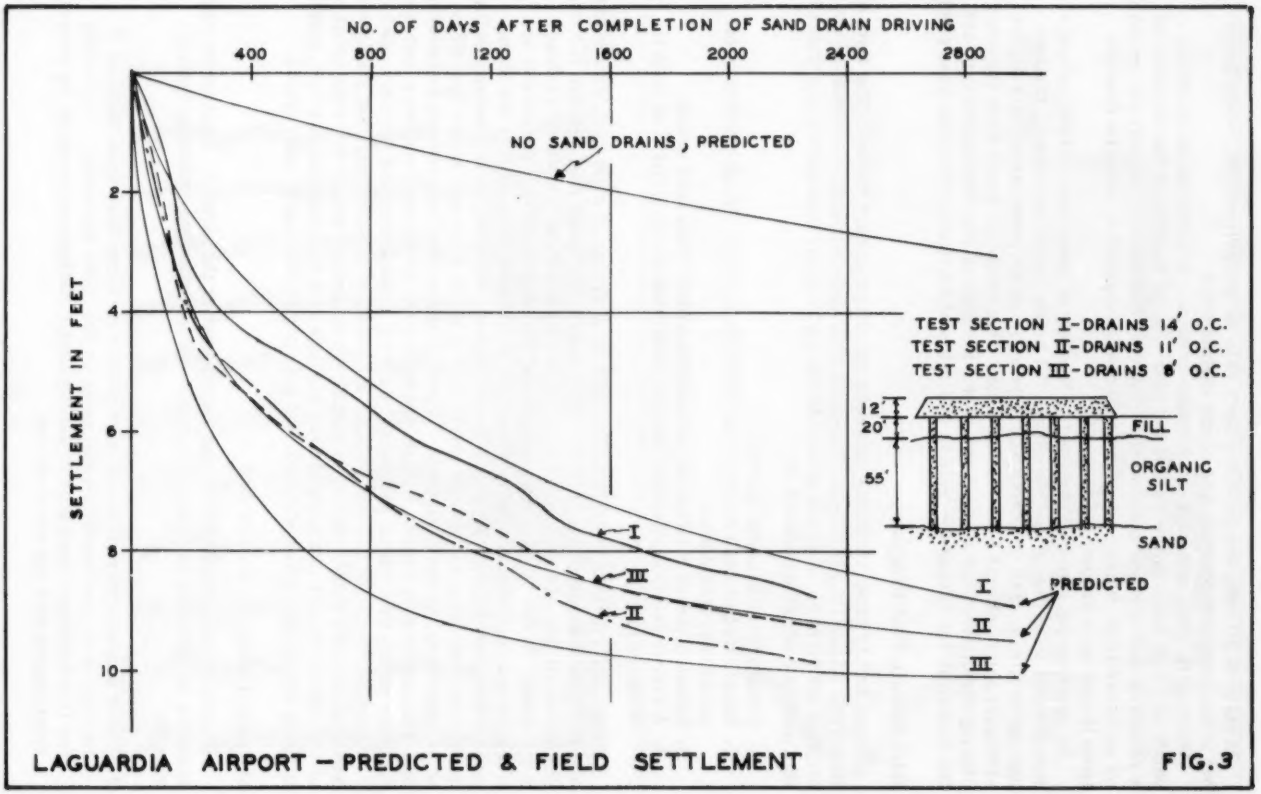
Three test sections were installed with sand drains at eight foot, eleven foot and fourteen foot spacings. A twelve foot surcharge (1200 lbs. per. sq. ft.) was placed on all test sections and continuous settlement and alignment readings were made. In the latter stages of placing the surcharge fill, the rate of application was speeded up. Note was made of a slight movement in the alignment stakes and filling operations were stopped. Continuous alignment readings showed no additional movement and in a very short time surcharge operations were started again and completed. Figure 3 shows the Test Section Installation and field settlement curves.[3] Also shown are the predicted settlement curves together with a theoretical settlement curve if sand drains had not been used.

Although results of the test section did not show conclusively that sand drains would accelerate consolidation as quickly as one might desire for rehabilitation of the entire airport in a very short time, one fact was certain; the soft, weak, organic silt gained shear strength and stability by the use of sand drains.

Physical Characteristics of Soil

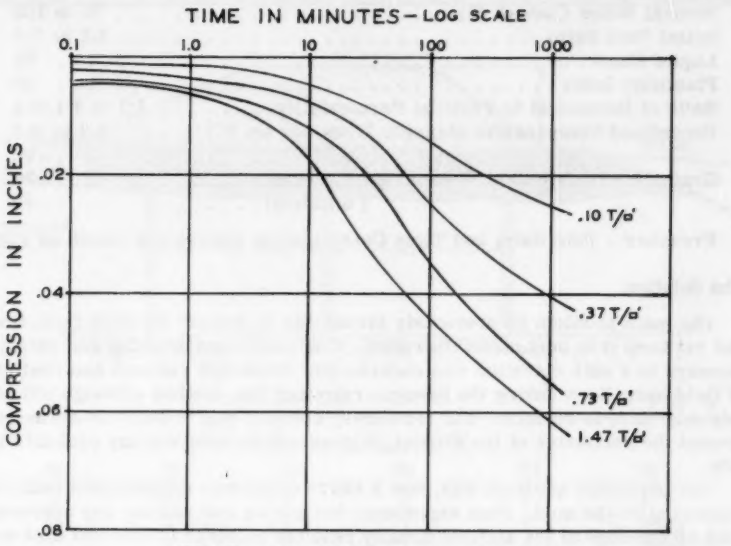
Extensive soil investigation was carried out at the airport. Laboratory tests made on undisturbed samples of organic silt indicated the following typical results:

3. There was an apparent interaction of the surcharge load upon Segment II because it is surrounded on two sides by the other segments. Correcting for this additional load moves the Segment II settlement curve to its proper relationship with the other curves.

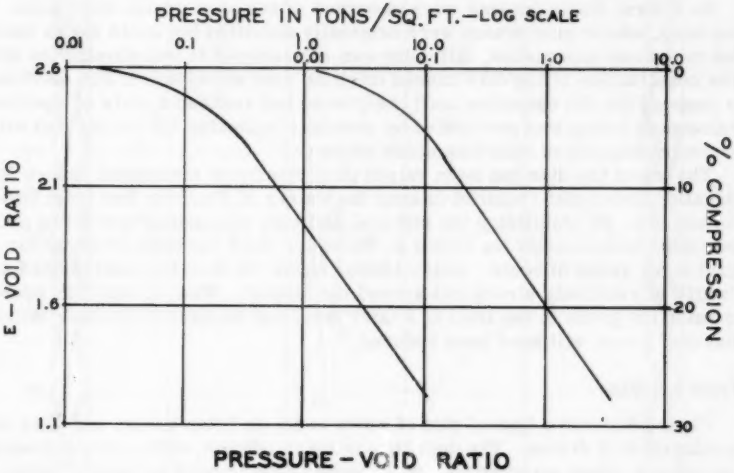


LAGUARDIA AIRPORT – PREDICTED & FIELD SETTLEMENT

FIG. 3



TIME – COMPRESSION CURVES



PRESSURE – VOID RATIO

LAGUARDIA AIRPORT

FIG. 4

Natural Water Content (%)	75 to 108
Initial Void Ratio	2.2 to 3.0
Liquid Limit	91
Plasticity Index	48
Ratio of Horizontal to Vertical Permeability	1:1 to 1 1/2:1
Unconfined Compressive Strength (Tons per sq. ft.)	0.2 to 0.6
Grain Size Distribution, % Passing (0.06mm	94
(0.02mm	86
(0.002mm	40

Pressure - Void Ratio and Time Consolidation Curves are shown on Figure 4.

The Solution

The main problem as previously stated was to protect the field from flooding and yet keep it in continuous operation. Complete sand draining and raising the runways to a safe elevation was considered. Economic reasons and limitation of field operations during the process ruled out this method although ultimately this may have to be done. The temporary solution was to build an earth dike around the perimeter of the airport, high enough to keep out any probable high tide.

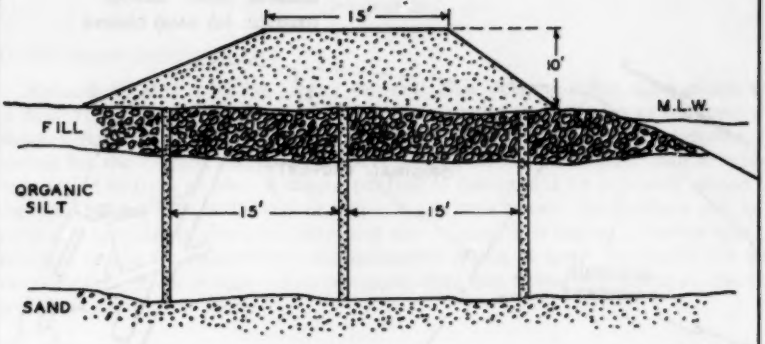
The important question was, how a heavy earth dike embankment could be supported on the mud. Past experience had shown that placing any appreciable load on the edge of the airfield usually resulted in shear failure and mud waves. The answer was obvious. Use sand drains in those areas where tests showed the subsoil not strong enough. Figure 5 shows a typical cross section of the dike where sand drains were used. Dike fill was placed in shallow lifts and measurements made of pore pressure and settlement.

No lateral displacements were observed where sand drains were used. In one area, where sand drains were originally specified but could not be installed due to runway approaches, difficulty was encountered in maintaining the dike. The construction of the dike caused offshore mud waves and it was necessary to suspend the fill operation until the ground had reached a state of equilibrium. Subsequent filling was performed by carefully balancing the fill on land with a corresponding fill of less magnitude offshore.

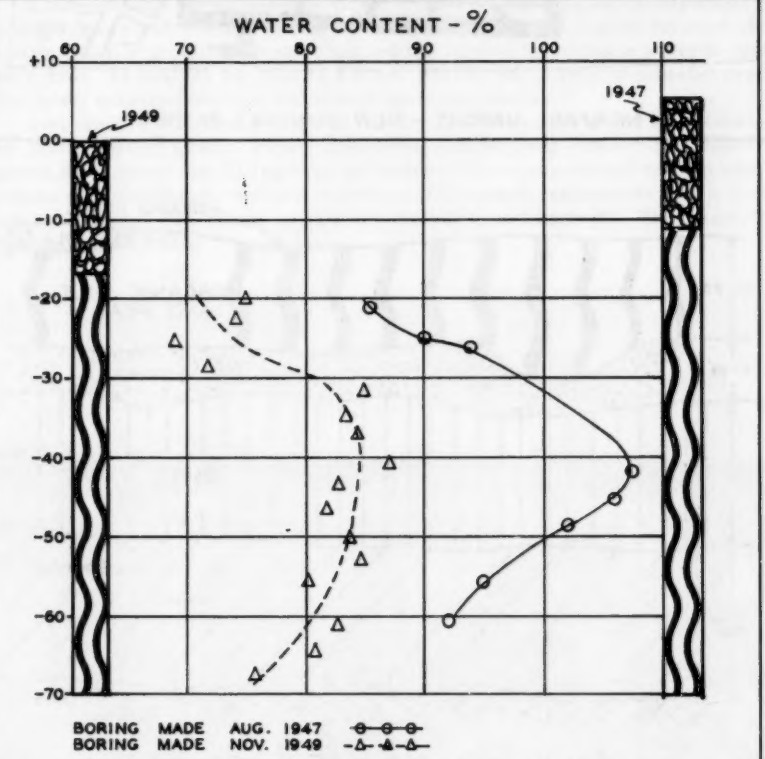
The top of the dike has been raised periodically as settlement occurs to an elevation above that required to keep the waters of Flushing Bay from flooding the airport. By stabilizing the soft soil and making construction of the dike possible, the necessity for filling in the entire field has been deterred for a good many years to come. Consolidation under the dike has also formed a "wall" of relatively strong silt around the airport. When it becomes necessary to raise the grade of the field at a later date, the possibility of shear failure and mud waves will have been reduced.

Field Results

Figure 6 shows a typical plot of water contents taken before and after consolidation with drains. The decrease in water content, with a corresponding increase in shear strength and the recorded drop in pore pressure, leaves no doubt in the authors' minds that the experiment in soil strangthening and consolidation acceleration has been successful.

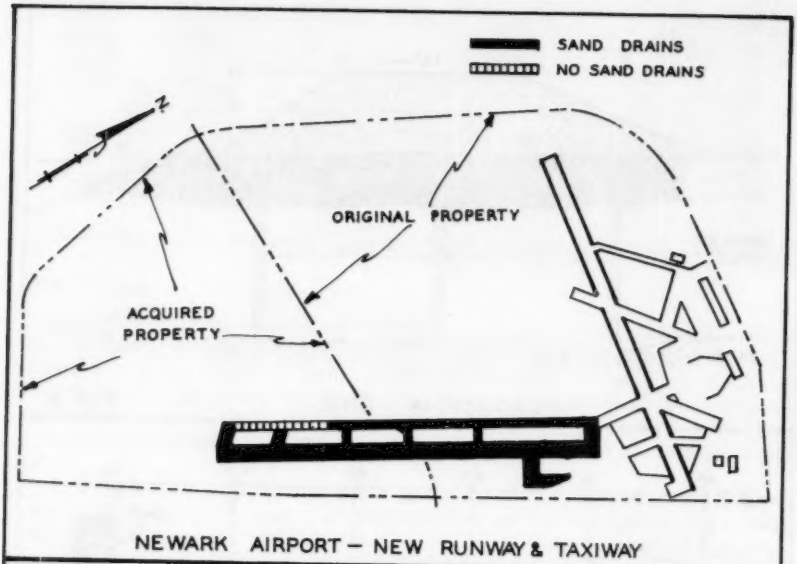


LAGUARDIA DIKE FIG. 5

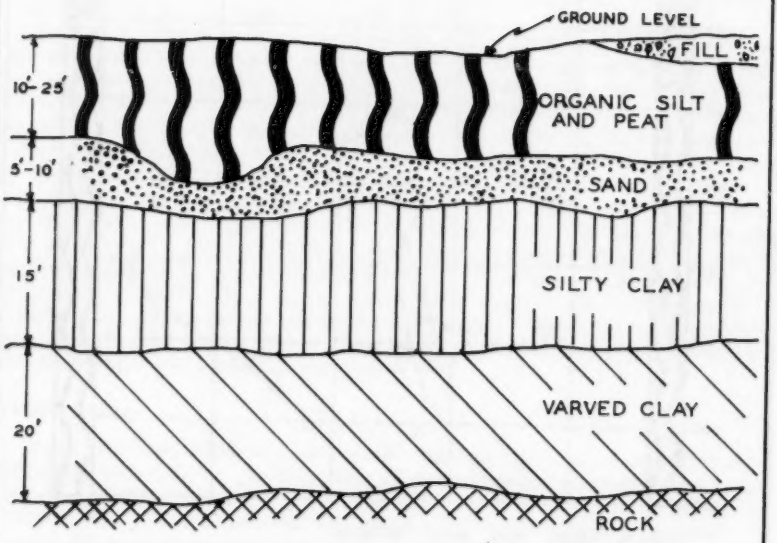


BEFORE AND AFTER CONSOLIDATION

LAGUARDIA AIRPORT FIG. 6



NEWARK AIRPORT – NEW RUNWAY & TAXIWAY



SOIL PROFILE – CENTER LINE OF NEW RUNWAY

FIG. 7

NEWARK AIRPORT
(Project 2)

Location and Geology of Site

Newark Airport is in the city of Newark, New Jersey, about eight miles west of New York City. The airport is located in a tidal marsh area to the west of Newark Bay. The topography is low and flat, typical of a marshland deposit, except for the original airport property which, for the most part, had previously been filled to final grade. A major portion of the expansion property added to the field by the Port Authority is below mean high water. Subsurface soil is a very soft and highly compressible peat and organic silt deposit. Below this stratum is found a relatively incompressible strata of sand, inorganic silt and varved clay. Rock is approximately sixty-five feet below the surface. Geologic profile is shown in Figure 7.

The Problem

Expansion and development of Newark Airport necessitated construction of a new runway and associated taxiways. Prevailing winds and the location of nearby heavily populated areas required the runway to extend in a North-South direction. Almost 75 per cent of the new runway would thus be situated over the newly acquired unfilled marshland described above.

A minimum of 10 feet of fill was required to bring the runway and taxiways up to final design grade. Prior experience with existing runways and paved areas have shown that filling over the unconsolidated marshland without some means of stabilization resulted in serious differential settlements which could not be tolerated on the new runway and taxiway (see Figure 8). Therefore, the problem was twofold:

Figure 8. Differential Settlement on Old Taxiway at Newark Airport

1. To support at least 10 feet of fill on the very weak virgin marshland.
2. The prevention of future settlement of the runways and taxiways after application of heavy fill loads necessary to build the runway and taxiway embankments to grade.

Time was of major importance for this project as a deadline had been set for opening of the runway and the design had to insure that time requirement.

Physical Characteristics of Soil

Borings were taken along the centerline of proposed runways and taxiways and all soil samples were classified and tested in the Port Authority laboratory. Typical results for the fine grained organic silt were:

Natural Water Content (%)	60 to 110
Initial Void Ratio	2.6 to 4.6
Liquid Limit	93
Plasticity Index	55
Ratio of Horizontal to Vertical Permeability	1:1 to 2:1
Unconfined Compressive Strength (Tons per sq. ft.)	0.1 to 0.3
Grain Size Distribution, % Passing (0.06mm	90
Grain Size Distribution, % Passing (0.02mm	70
Grain Size Distribution, % Passing (0.002mm	30

Pressure - Void Ratio and Time Consolidation Curves are shown on Figure 9.

Here again we find a weak soil having very poor drainage characteristics. The sand below the compressible soil had too much silt for proper drainage and so it was necessary to consider the organic silt as having a single drainage face only at the top where it came in contact with the proposed porous fill blanket.

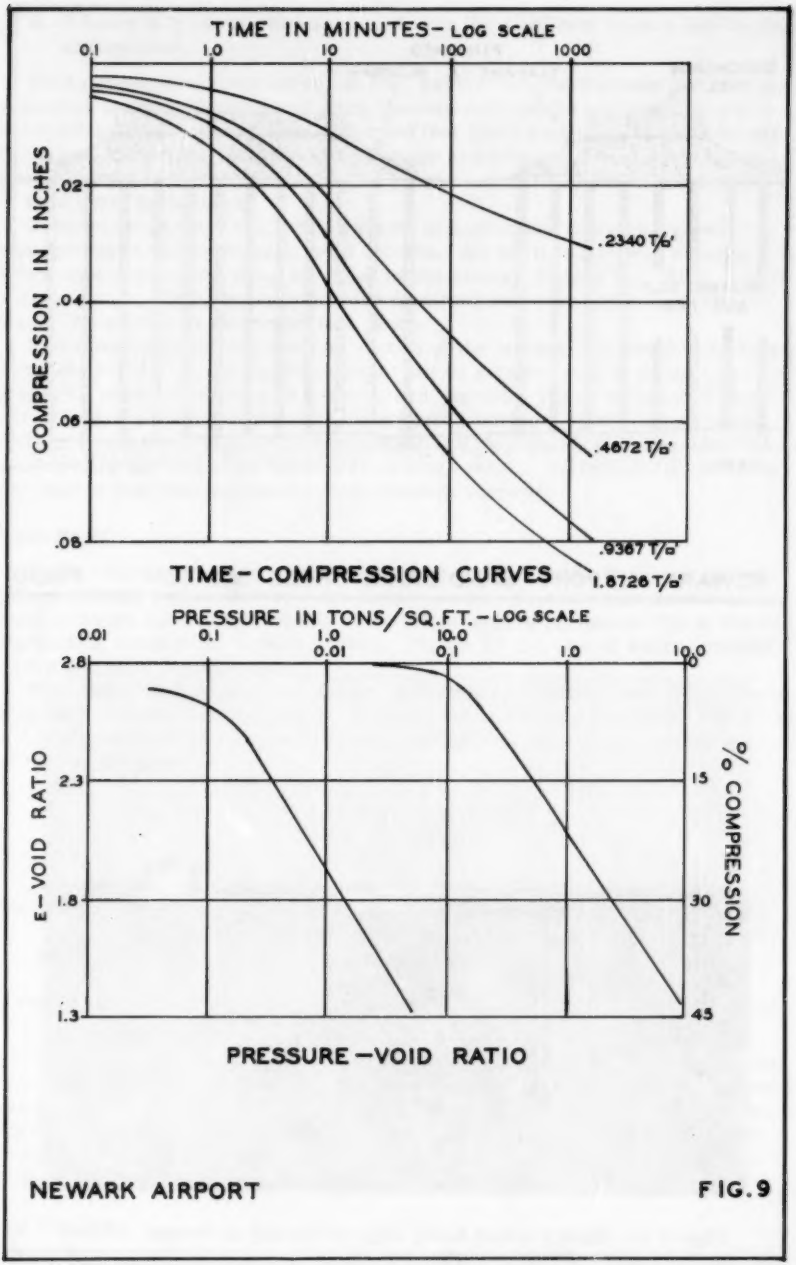
The Solution

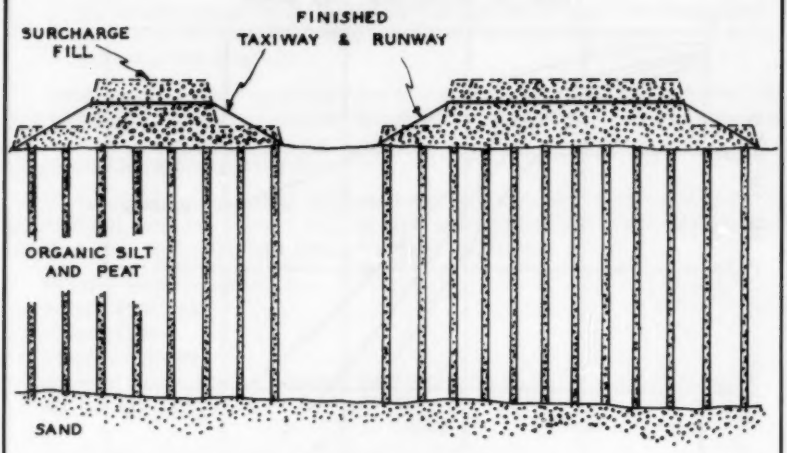
Many possible methods of construction were considered. Some of them were:

1. Excavate mud and backfill with granular soil.
2. Support runway on piles.
3. Consolidate mud with surcharge load only.

All of these methods were dropped for either economic or time factors. With the experience gained at La Guardia Airport it was decided to use sand drains to accelerate consolidation and help strengthen the silt. One reason that the use of sand drains and surcharge fill proved so economical was that the excess surcharge fill could be completely used for the required runway shoulders and side slopes without waste. Construction procedure was as follows:

1. A specially selected porous sand blanket was placed on the areas to be occupied by the runway and taxiway.
2. Twenty inch diameter sand drains were installed by means of a closed mandrel through the blanket, the compressible peat and organic silt and into the firm stratum underlying the meadow.
3. Porous drain pipes were installed in the sand blanket and terminated in drainage ditches along the side of runway.
4. Required fill and additional surcharge load (1200 lbs. per sq. ft.) were placed in specified lifts.
5. After consolidation had taken place and piezometers and settlement plates indicated no substantial additional settlement could be expected, the surcharge fill was moved to the sides to form the necessary shoulders and side slopes.


TIME IN MINUTES – LOG SCALE
0.1
1.0
10
100
1000
COMPRESSION IN INCHES
.02
.04
.06
.08
.2340 T/□'
.4672 T/□'
.9367 T/□'
1.8726 T/□'
TIME – COMPRESSION CURVES
PRESSURE IN TONS/SQ.FT. – LOG SCALE
0.01
0.1
1.0
10.0
0.01
0.1
1.0
10.0
E – VOID RATIO
2.8
2.3
1.8
1.3
% COMPRESSION
15
30
45
PRESSURE – VOID RATIO
NEWARK AIRPORT
FIG. 9



NEWARK AIRPORT - SAND DRAIN CROSS SECTION FIG.10

Figure 11. Slide Failure Along Edge of Runway at Newark Airport

6. A heavy duty pavement was placed over the stabilized runway and taxiway embankment.

Sand drains were installed on ten feet, twelve feet and fourteen feet centers, depending upon the thickness of compressible soil and its permeability characteristics. Final observations indicated that there was very little difference in the ten, twelve and fourteen foot spacings and a twelve foot spacing could probably have been used for the entire project. Figure 10 shows a cross section of sand drain installation.

In three localized areas, when the rate of application of surcharge weight was increased and a hydraulic head condition due to fill water was created, minor slides occurred along the sides of the runway (Figure 11). After a short waiting period, filling operations were continued and very close observations made. No additional movement took place.

It is interesting to note that one section of the taxiway was found to be almost completely underlaid by a peaty organic silt or meadow mat, a highly compressible material made up of a network of vegetable fibres mixed with some silt. Consolidation tests showed this material, having a water content up to 400 per cent, to be very permeable and for that reason, sand drains were deemed unnecessary and therefore omitted from this section. Surcharge fill, however, was placed over this section for consolidation purposes.

Field Results

Typical field settlement curves are shown on Figure 12. In all cases, surcharge fill was removed after a six month period. No noticeable rebound was found although there was probably some slight upward movement due to the soil readjusting itself to the lighter loading. Figure 13 is a plot of water contents before and after consolidation.

The runway and taxiway has been in operation since November 1952. There is no sign of settlement or lateral displacement nor is any expected. Figure 14 is a photograph of the completed runway and taxiway with the surrounding marshland terrain.

PORT NEWARK
(Project 3)

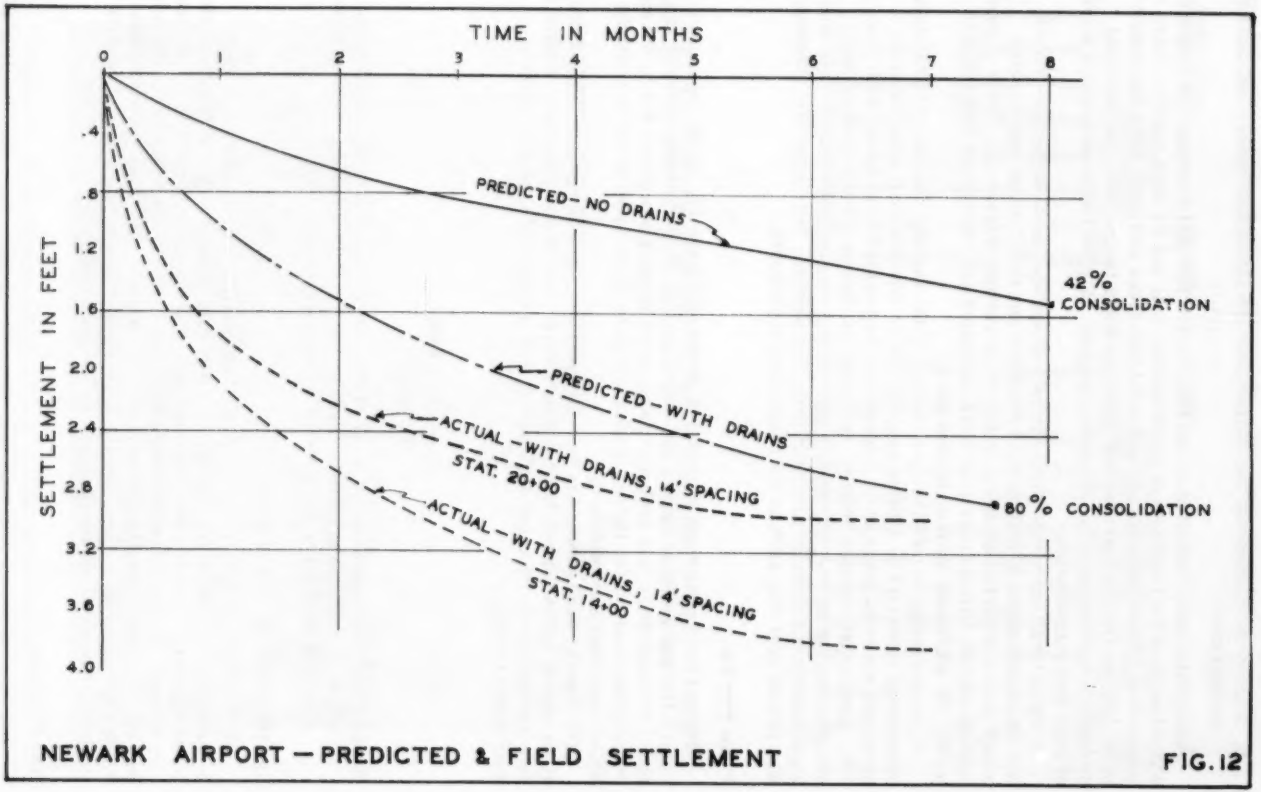
Note: A more comprehensive study of the use of sand drains at Port Newark may be found in the Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering.

Location and Geology of Site

Port Newark is located in the same tidal marsh area as Newark Airport. It consists of a man-made channel leading from Newark Bay and is surrounded by warehouses, transit sheds, storage areas, etc. Land adjacent to the channel was brought up to its present grade by filling over the low marshland with muck and clay dredged from the channel. Below the marsh is found the same relatively incompressible sand, inorganic silts and clays as was found at Newark Airport. Figure 15 shows geologic profiles for this area.

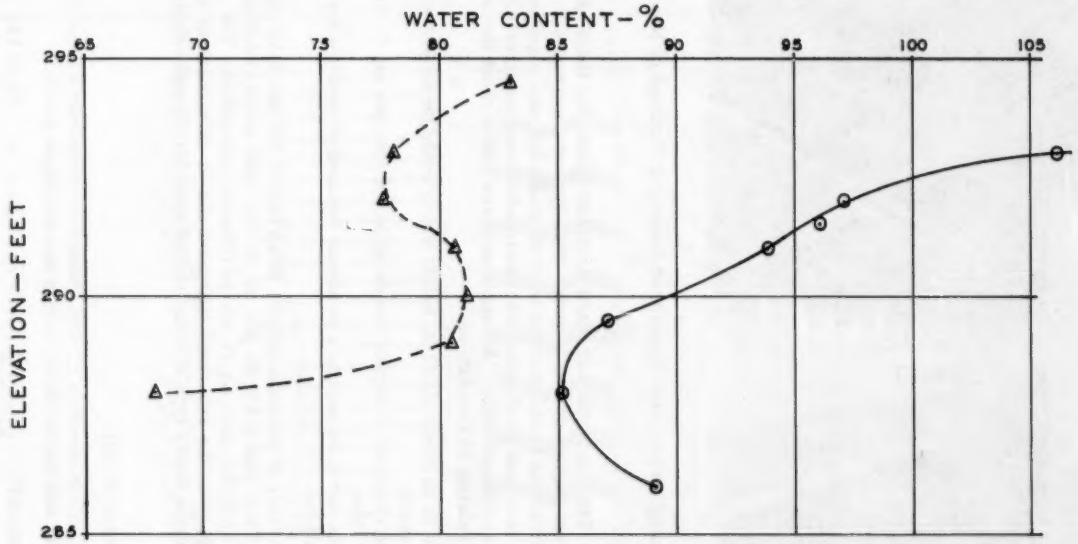
The Problem

The Port Authority assumed responsibility for operating, rehabilitating and developing Port Newark in 1947, and immediately constructed two transit sheds,



NEWARK AIRPORT — PREDICTED & FIELD SETTLEMENT

FIG. 12



EFFECT OF CONSOLIDATION ON ORGANIC SILT

NEWARK AIRPORT

FIG. 13

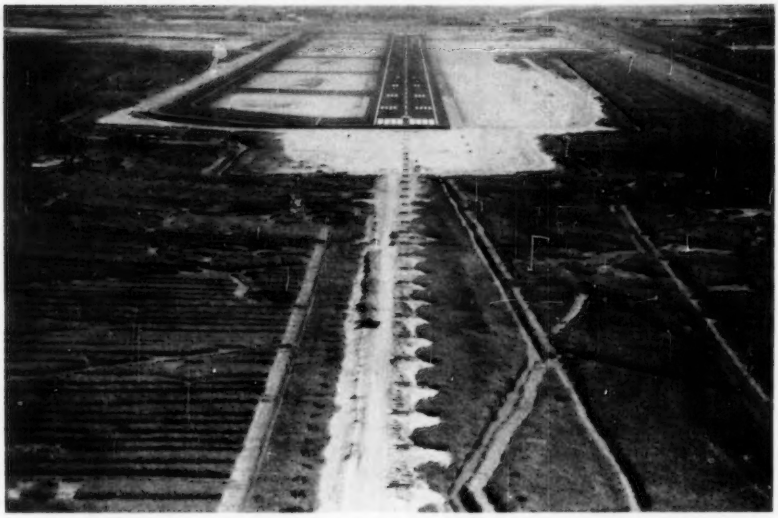


Figure 14. Surrounding Terrain and Completed Runway & Taxiway at Newark Airport

A and B (Figure 15). The floor slab and concrete grade beams for these sheds were supported on steel H piles driven to rock. This type of construction eliminated any complications because of the poor subgrade but was expensive.

In 1951, the heavy increase in shipping warranted additional construction of one warehouse and five transit sheds. All six structures had to meet three major requirements relating to foundations.

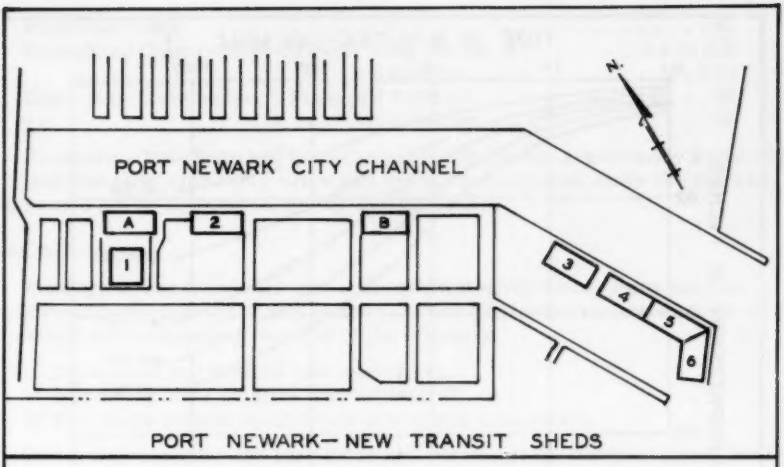
1. The floor had to be at truck platform height which would be three feet above existing grade.
2. Floors would be required to support loads up to 600 lbs. per sq. ft. for long periods of time.
3. Floor settlement had to be kept to a minimum because of tenant's fork-lift truck operations.

In order to put the floor at platform height, three feet of fill had to be placed above ground level. This, plus a 600 lb. per sq. ft. live load would result in placing an additional 1000 lbs. per. sq. ft. on the filled in marshland. The problem was how to prevent the expected settlement and/or displacement without reverting to the same costly type of construction used for Transit Sheds A and B.

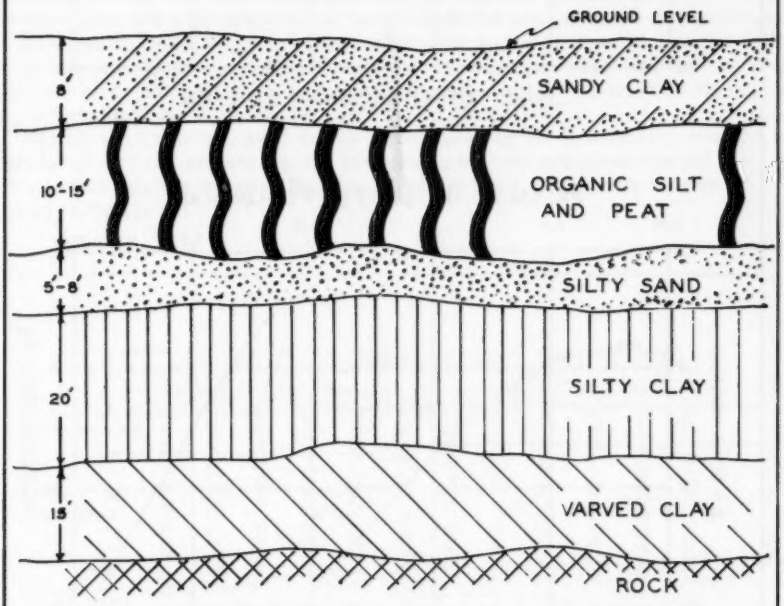
Physical Characteristics of Soil

The compressible soil in question at Port Newark is of the same deposit as that of Newark Airport and the following values are therefore similar:

Natural Water Content (%)	60 to 125
Initial Void Ratio	1.5 to 2.7
Liquid Limit	82

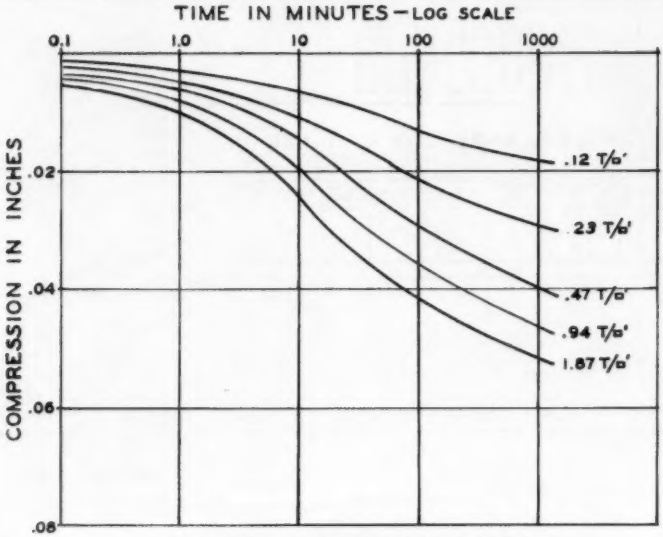


PORT NEWARK— NEW TRANSIT SHEDS

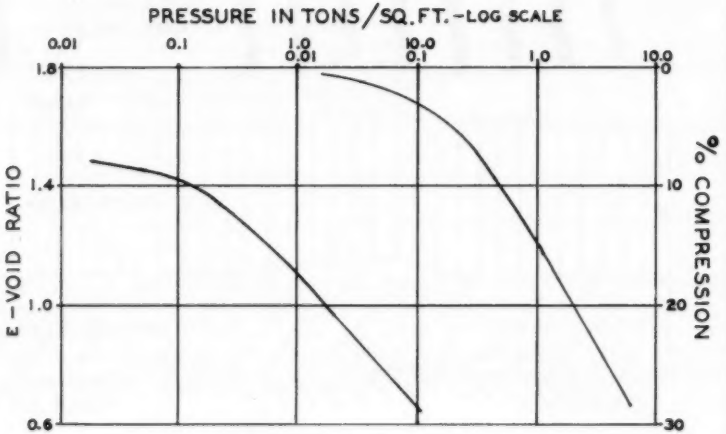


SOIL PROFILE - NEW TRANSIT SHED AREA

FIG. 15



TIME – COMPRESSION CURVES



PRESSURE – VOID RATIO

PORT NEWARK

FIG. 16

Plasticity Index	48
Unconfined Compressive Strength (tons per Sq. ft.)	0.4 to 0.5
Grain Size Distribution, % Passing (0.06mm	100
(0.02mm	86
(0.002mm	10

Pressure - Void Ratio and Time Consolidation Curves are shown in Figure 16.

Both the silty sand layer below and the sandy clay layer above the marsh soil could not be considered proper drainage faces.

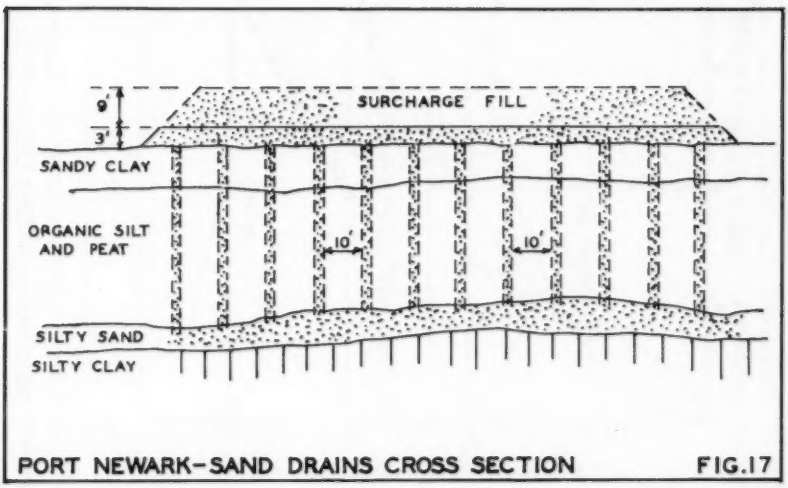
The Solution

Although to our knowledge this technique had never before been used for building construction, sand drains and surcharge fill were estimated to be more practical and economical than any of the following:

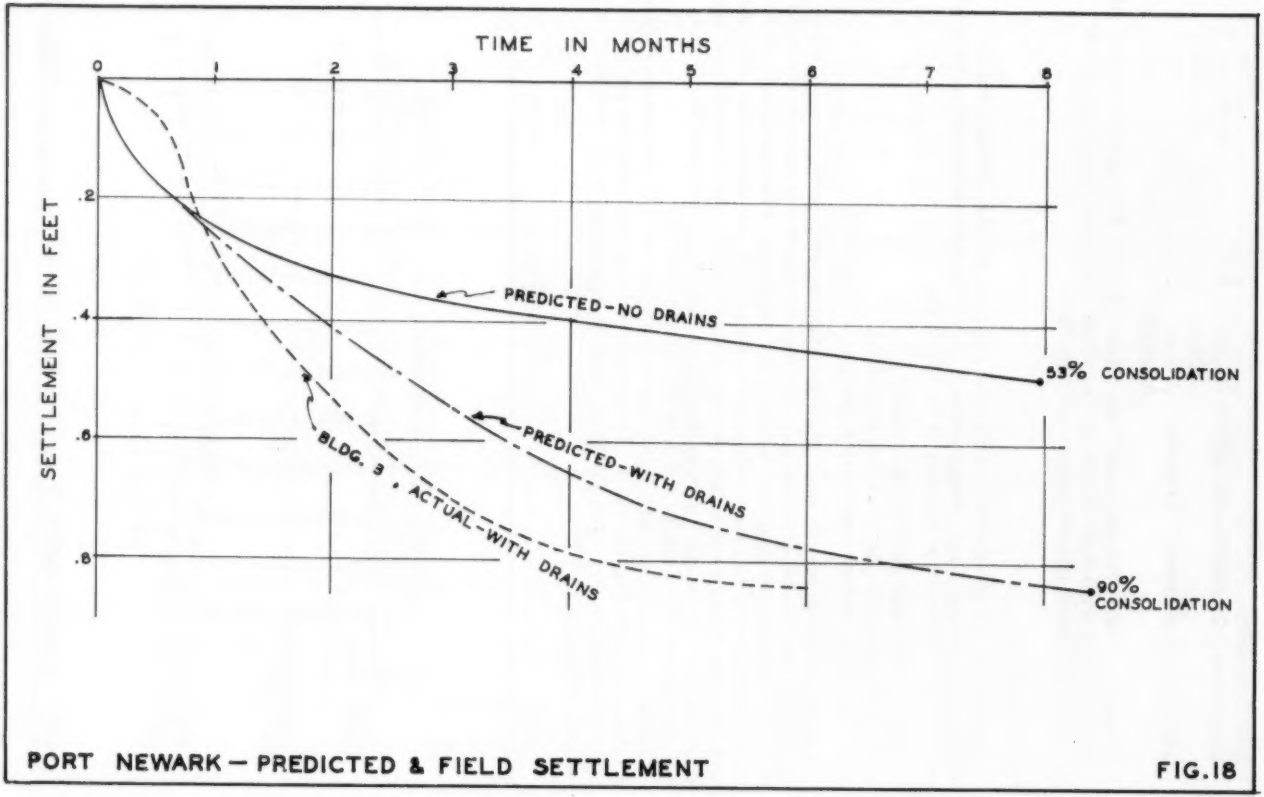
1. Excavation and backfill (too expensive).
2. Supporting floor on piles (too expensive).
3. Surcharge without sand drains (too long a time delay).

One main reason for the sand drain and surcharge fill being so much more economical than the other techniques was the fact that the surcharge fill could be used over and over again for at least four buildings. Planned construction of the first four buildings was staggered so that as the surcharge was removed from one site it was immediately placed on the next site which had already been sand drained. Surcharge fill from the last two buildings was used to fill in a bay area where the bulkhead had been moved 100 feet east.

For all six buildings, sand drains were placed on ten foot centers and had a surcharge of 1400 lbs. per sq. ft. Six months waiting was predicted but the time varied slightly for each building. Typical sand drain cross section is shown in Figure 17.

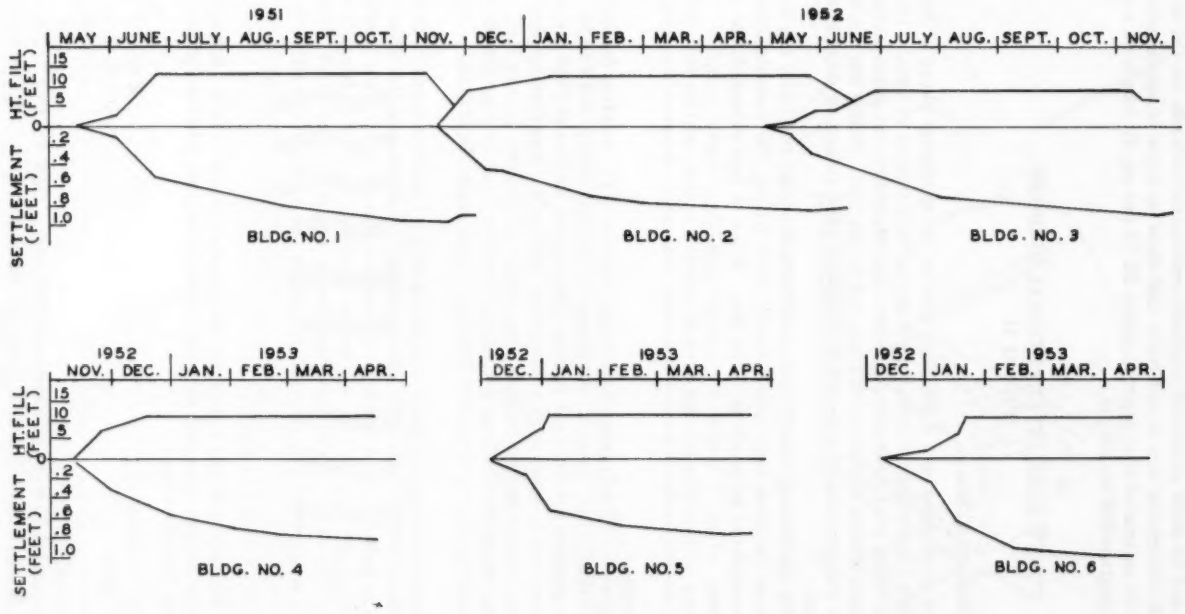


PORT NEWARK-SAND DRAINS CROSS SECTION FIG.17



PORT NEWARK — PREDICTED & FIELD SETTLEMENT

FIG. 18



PORT NEWARK - FIELD SETTLEMENT OF SIX BLDGS. FIG. 19

Field Results

Figures 18 and 19 show predicted and field settlement records for the six structures. All structures are in use today and show no signs of appreciable settlement. It is estimated that approximately $1.74 per sq. ft. has been saved by using this construction technique.

NEW YORK INTERNATIONAL AIRPORT
(Project 4)

Location and Geology of Site

This airport is situated on a 5,000 acre site on the northeast shore of Jamaica Bay, Long Island. More than 85 per cent of this area consists of swamp land which has been filled over hydraulically with sand dredged from adjacent Jamaica Bay. This swamp varies in thickness from one to ten feet. Below the marine deposits of the swamp area is a deep bed of glacial sand extending down over one hundred feet.

Before filling operations, which took place between 1942 and 1945, the top of the swamp was at the mean high water level. After filling, the compressible swampland consolidated as much as three feet. Some of this consolidation is still taking place.

Figure 20 is a soil profile for the area at which a hangar will be constructed.

The Problem

The rapid growth of air transportation has overtaxed the existing facilities at New York International Airport. Additional terminal, cargo, and hangar structures are planned for the near future. During the middle of 1953 the Port Authority entered into a lease with a scheduled airline for construction and rental of a large hangar having an open floor space of 250,000 square feet.

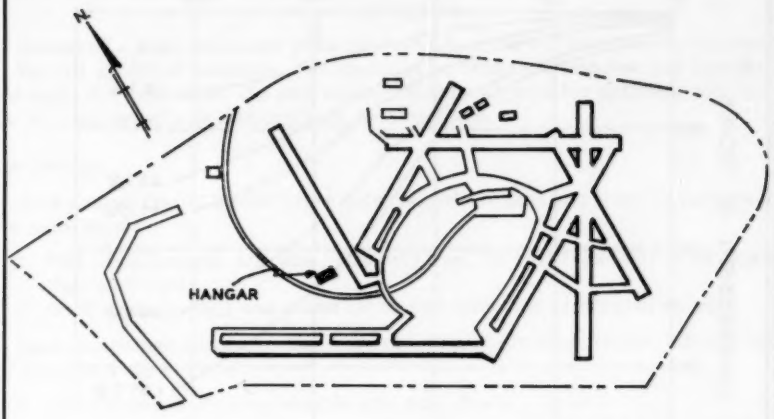
In the opinion of the mechanical engineers, radiant floor heating for the type of hangar planned was more desirable than any other type. Here then, we have the problem.

How to construct a concrete floor containing radiant heating pipes over the filled in swampland described above without inducing cracking of the floor and consequent leaks in the radiant heating system at a future date. This problem was aggravated all the more by the necessity of adding two feet of fill to bring the hangar floor to grade. Experience had shown, in previous smaller structures, that differential settlement will occur, when loads are added to the filled in land.

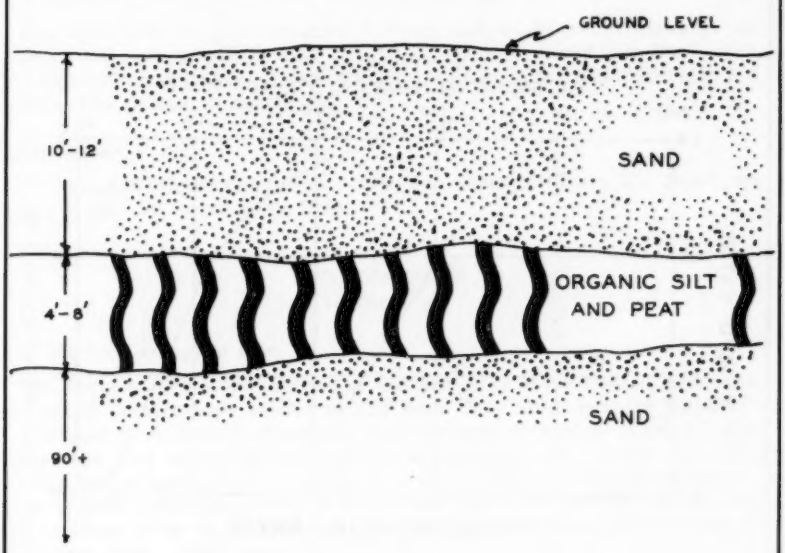
Physical Characteristics of the Soil

Here again we have a highly compressible organic meadow soil. Borings were made in the hangar area and samples brought into the laboratory for testing. Typical results were:

Natural Water Content (%)	78 to 110
Initial Void Ratio	1.7 to 2.5
Liquid Limit	109
Plasticity Index	49
Ratio of Horizontal to Vertical Permeability	1:1 to 1 1/2:1
Unconfined Compressive Strength (tons per sq. ft.)	0.5 to 0.6

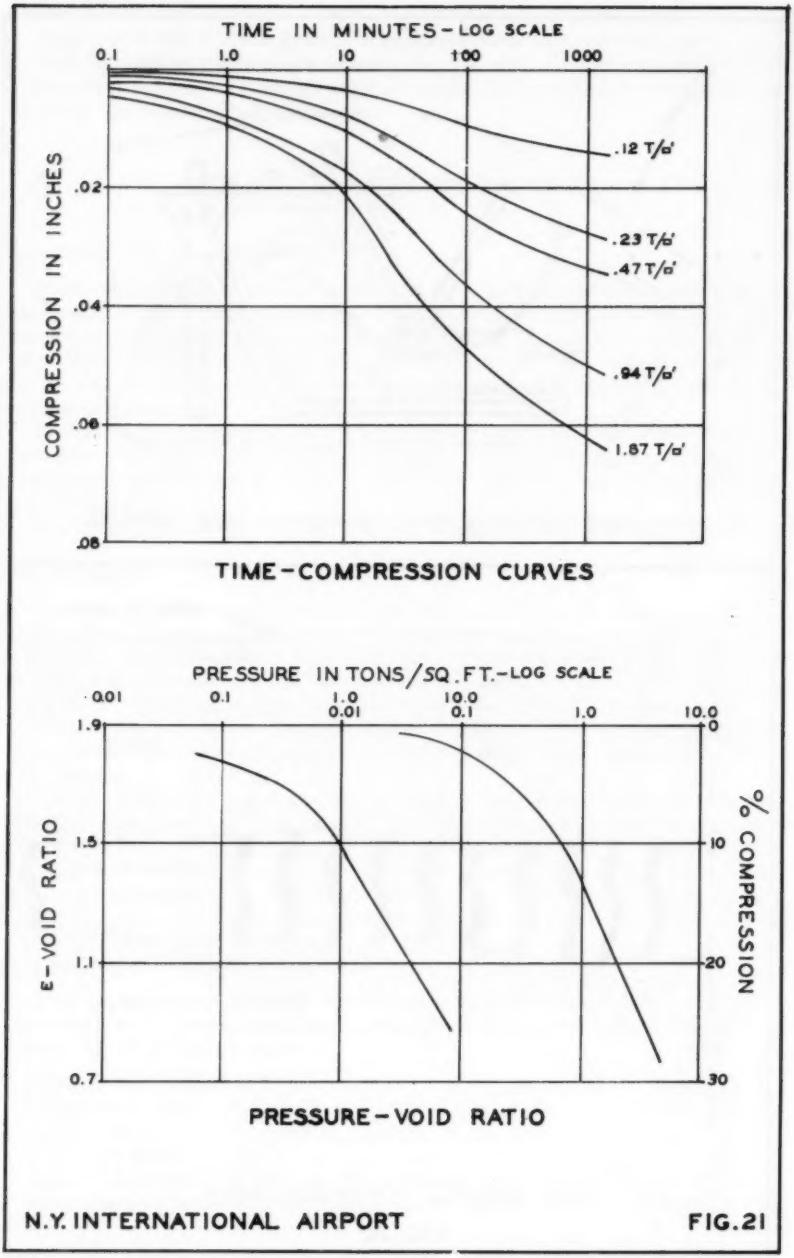


NEW YORK INTERNATIONAL AIRPORT - NEW HANGAR



SOIL PROFILE — HANGAR AREA

FIG. 20


TIME IN MINUTES - LOG SCALE
0.1
1.0
10
100
1000
COMPRESSION IN INCHES
.02
.04
.06
.08
.12 T/□'
.23 T/□'
.47 T/□'
.94 T/□'
1.87 T/□'
TIME - COMPRESSION CURVES
PRESSURE IN TONS/SQ. FT. - LOG SCALE
0.01
0.1
1.0
10.0
0.01
0.1
1.0
10.0
E - VOID RATIO
1.9
1.5
1.1
0.7
% COMPRESSION
0
10
20
30
PRESSURE - VOID RATIO
N.Y. INTERNATIONAL AIRPORT
FIG. 21

Grain Size Distribution, % Passing	(0.06mm	100
	(0.02mm	95
	(0.002mm	20

Pressure - Void Ratio and Time Consolidation Curves are shown in Figure 21.

For all practical purposes, the glacial sand below the meadow mat and the hydraulic fill sand above the mat can be considered good for drainage and this can be considered a double drainage face problem.

The Solution

This project was different from both Newark Airport and Port Newark on two main points:

1. The compressible soil was relatively thin, ranging from four to no more than eight feet.
2. Good drainage soil was found above and below the compressible soil.

Sand drains and surcharge fill were considered for this project, but not used for the following reasons:

1. The thickness of compressible soil was small.
2. Laboratory consolidation tests indicated a reasonable short time required for 80 to 90 per cent consolidation under a surcharge load without sand drains.
3. The good draining granular sand faces above and below the compressible mat.

For these reasons, the use of sand drains was not deemed necessary. Instead a twelve foot surcharge fill (1200 lbs per sq. ft.) was placed over the hangar area in October and contracts awarded for building construction to begin six months after placing of surcharge.

Field Results

Field and predicted settlement curves are shown in Figure 22. Surcharge fill was removed during April 1954.

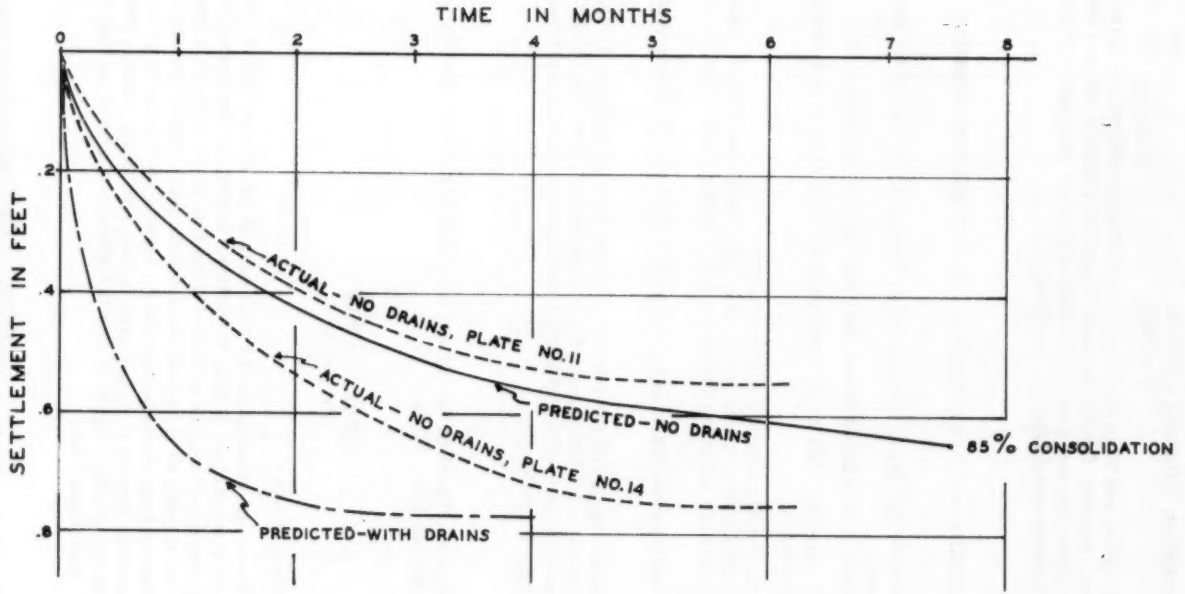
HOBOKEN PIERS
(Project 5)

Location and Geology of Site

The Hoboken Piers are situated on the west bank of the Hudson River just opposite of the mid-town section of New York City. Outside of the bulkhead line is a normal river deposit of marine silts and some fine sand. This is a deep deposit but thins out quickly as it passes the bulkhead line. Below the organic silt is a bed of sand resting on serpentine rock. Inshore of the bulkhead is filled in land consisting mostly of mixed sand, ashes and debris resting over a thinner layer of the river silt. Again, below the silt is some sand resting on serpentine rock. (See Figure 23.)

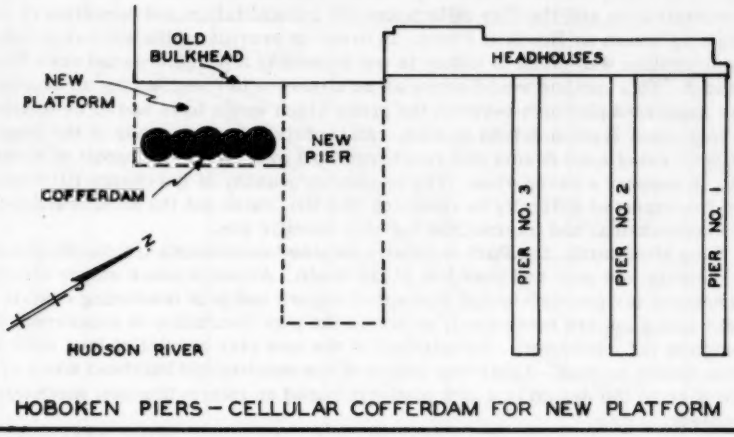
The Problem

Piers originally constructed by the North German Lloyd & Hamburg American Steamship Lines were taken over by the United States Government during World War I and operated from then on first by the U.S. Army and then by the Maritime

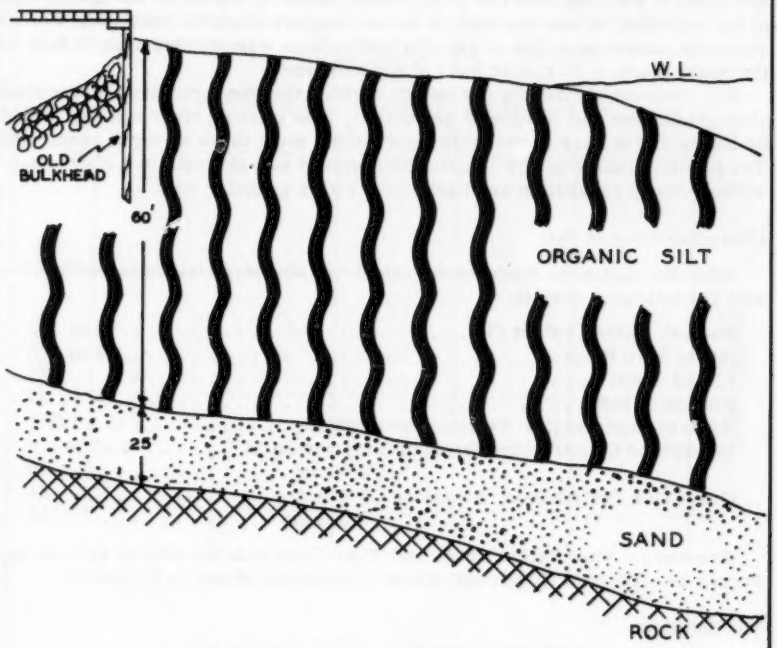


N.Y. INTERNATIONAL AIRPORT - PREDICTED & FIELD SETTLEMENT

FIG. 22



HOBOKEN PIERS – CELLULAR COFFERDAM FOR NEW PLATFORM



SOIL PROFILE – NEW PLATFORM

FIG. 23

Administration. Prior to and during World War II, fire destroyed three of the piers. In 1952, the Port Authority entered into an agreement with the Maritime Administration and the City of Hoboken for rehabilitation and operation of the property known as Hoboken Piers. In order to provide additional cargo space, consideration was given to filling in and providing a paved area between Piers 2 and 3. This method would serve as an alternate to constructing an expensive pile supported platform between the piers which would have had to be founded on long steel H piles driven to rock. An investigation was made of the feasibility of using sand drains and surcharged fill over the deep deposit of river silt, to support a paved area. The enormous quantity of surcharge fill required and the expected difficulty in retaining this fill, ruled out the sand drain technique as uneconomical and impractical for this specific job.

Soon afterwards, the Port Authority retained consultants for the design of an entirely new pier and platform to the South. Although some unique structural techniques for pre-tensioning precast stringers and post tensioning cast-in-place pile caps are being used, as far as the pier foundation is concerned, the design is not uncommon. Foundations of the new pier consist of long steel H piles driven to rock. Little was known of the existing old bulkhead south of the new pier so the design of a new platform posed an interesting soil mechanics problem.

No construction records could be found for the design of the old bulkhead and since it was expected that heavy loads would be placed on the upland back of the bulkhead, it was felt that additional support would be required. For construction purposes, a line of cellular cofferdams was decided upon to hold back the heavy loads to be placed back of the bulkhead.

For reasons of stability the design of the cofferdams required a specified strength for the soil in back of and within. The existing river silt that would be inside and in back of the cofferdam did not meet those strength requirements. The problem than was how to gain the required soil strength in a short time without costly excavation and backfilling with a granular soil.

Characteristics of Soil

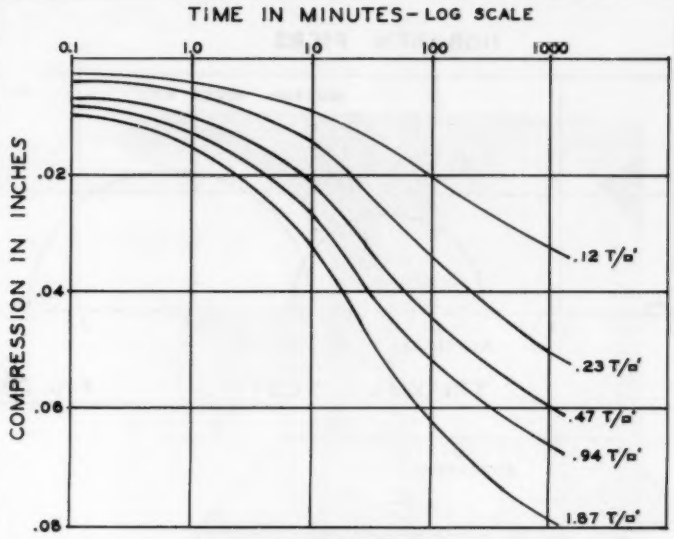
After borings were made, undisturbed samples were tested in the laboratory with the following results:

Natural Water Content (%)	60 to 139
Initial Void Ratio	1.2 to 3.5
Liquid Limit	75
Plasticity Index	33
Ratio of Horizontal to Vertical Permeability	1:1 to 1 1/2:1
Unconfined Compressive Strength (tons per sq. ft.)	0.05 to 0.5
Grain Size Distribution, % Passing (0.06mm	90
(0.02mm	64
(0.002mm	20

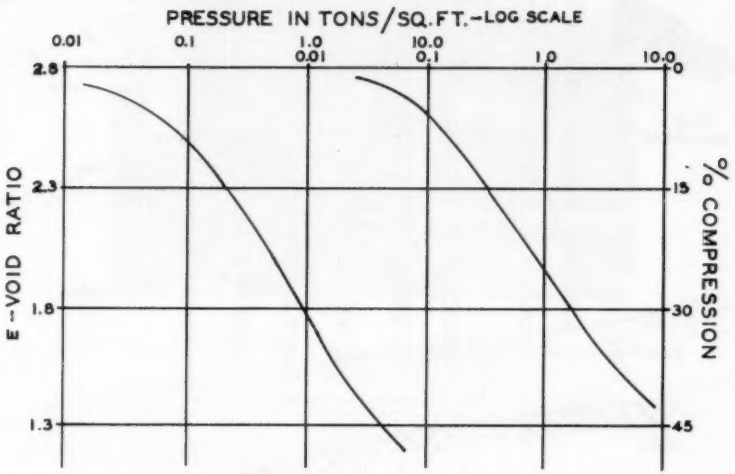
Pressure - Void Ratio Curves and Time Consolidation curves are shown in Figure 24. Triaxial Compression test results are shown in Figure 25.

The Solution

As previously stated, the problem for this project was obtaining a certain strength soil in and behind the cofferdam to insure stability. One method would be to excavate the river silt and replace it with sand. However, this was too costly and meant additional internal cross bracing for the cofferdams.



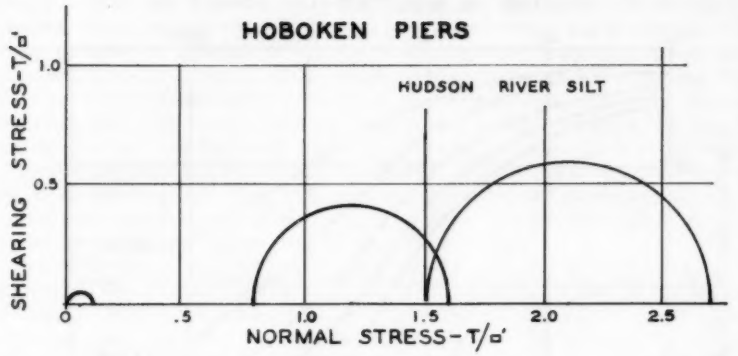
TIME - COMPRESSION CURVES



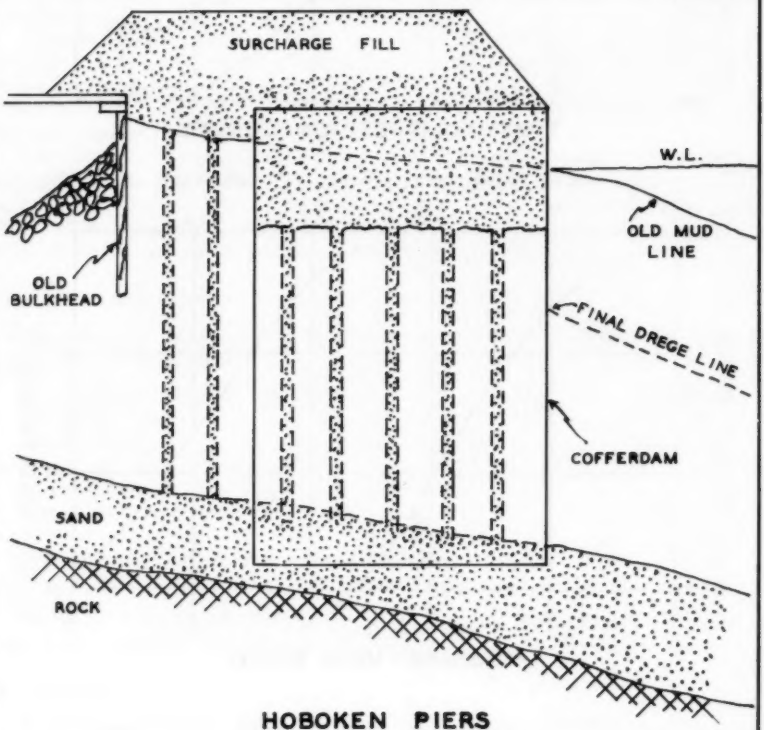
PRESSURE - VOID RATIO

HOBOKEN PIERS

FIG. 24



TRIAXIAL TEST FIG. 25



HOBOKEN PIERS

SAND DRAINS-CROSS SECTION FIG. 26

Figure 27. Driving Sand Drains in Cofferdam at Hoboken Piers

Investigation of Triaxial test results indicated that the silt would have the necessary shear strength when consolidated to a certain degree. This increase in shear strength was all that was needed for stability of the cofferdams.

Because of the thickness of river silt and the tight construction schedule, sand drains were used to reach the degree of consolidation required as soon as possible. They were also installed inshore of the cofferdams as that area would eventually be required to support a pavement.

A ten foot surcharge load (1000 lbs per sq. ft.) was placed over the entire sand drained area. Sand drain layout is shown in Figure 26 and field installation in Figure 27.

Field Results

At the time this paper was submitted, surcharge fill had not as yet been removed. Field results will be presented when the paper is delivered at the June 1954 meeting in Atlantic City and submitted as a closing to this paper.

CONCLUSION

A condensed history has been presented of five engineering projects for which sand drains were considered as an aid to construction. In some cases they were used, in others they were not. The projects are summarized in Figure 28.

Reading up to this point, one will surmise that we advocate the use of sand drains. In a sense we do. We feel that in many cases they serve as excellent tools. However, as in the case of the hangar site, there are instances where they are not at all necessary and would only increase costs.

There may also be certain instances where remolding due to sand drain installation may weaken a soil temporarily and cause problems. However, any tool can be misused and good engineering judgment should make those instances very rare indeed.

PROJECT	PURPOSE OF SAND DRAINS	PERCENT OF PROJECT SAND DRAINED (%)	THICKNESS OF UNSTABLE SOIL (FT.)	SPACING OF SAND DRAINS (FT.)	SURCHARGE TIME (MONTHS)
(1) LAGUARDIA AIRPORT	Increase Strength	40	55	15	*
(2) NEWARK AIRPORT	Increase Strength and Accelerate Consolidation	95	10 - 25	10, 12, 14	6
(3) PORT NEWARK	Accelerate Consolidation	100	10 - 15	10	4 - 6
(4) NEW YORK INTERNATIONAL AIRPORT	Accelerate Consolidation	0	4 - 8	No sand drains	6
(5) HOBOKEN PIERS	Increase Strength	100	60	9	6

* No surcharge used on this project. The dike embankment itself was used to consolidate and strengthen the organic silt.

REVIEW OF PORT AUTHORITY SAND DRAIN PROJECTS FIG. 28

PROCEEDINGS-SEPARATES

The technical papers published in the past twelve months are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

VOLUME 79 (1953)

JULY: [a] 200(SM)[b], 201(ST)[b], 202(EM)[b], 203(SM)[b], 204(AT)[b], 205(EM)[b], 206(ST)[b], 207(SA)[b], 208(SA)[b], 209(ST)[b], 210(SU)[b], 211(EM)[b], 212(SU)[b], 213(IR)[b], 214(HW)[b], 215(SM)[b], 216(ST)[b], 217(ST)[b], 218(ST)[b], 219(ST)[b], 220(SM)[b], 221(HW)[b], 222(SM)[b], 223(EM)[b], 224(EM)[b], 225(EM)[b], 226(CO)[b], 227(SM)[b], 228(SM)[b], 229(IR)[b].

AUGUST: 230(HY), 231(SA), 232(SA), 233(AT), 234(HW), 235(HW), 237(AT), 238(WW), 239(SA), 240(IR), 241(AT), 242(IR), 243(ST), 244(ST), 245(ST), 246(ST), 247(SA), 248(SA), 249(ST), 250(EM)[c], 251(ST), 252(SA), 253(AT), 254(HY), 255(AT), 256(ST), 257(SA), 258(EM), 259(WW).

SEPTEMBER: 260(AT), 261(EM), 262(SM), 263(ST), 264(WW), 265(ST), 266(ST), 267(SA), 268(CO), 269(CO), 270(CO), 271(SU), 272(SA), 273(PO), 274(HY), 275(WW), 276(HW), 277(SU), 278(SU), 279(SA), 280(IR), 281(EM), 282(SU), 283(SA), 284(SU), 285(CP), 286(EM), 287(EM), 288(SA), 289(CO).

OCTOBER: [d] 290(all Divs), 291(ST)[c], 292(EM)[c], 293(ST)[c], 294(PO)[c], 295(HY)[c], 296(EM)[c], 297(HY)[c], 298(ST)[c], 299(EM)[c], 300(EM)[c], 301(SA)[c], 302(SA)[c], 303(SA)[c], 304(CO)[c], 305(SU)[c], 306(ST)[c], 307(SA)[c], 308(PO)[c], 309(SA)[c], 310(SA)[c], 311(SM)[c], 312(SA)[c], 313(ST)[c], 314(SA)[c], 315(SM)[c], 316(AT), 317(AT), 318(WW), 319(IR), 320(HW).

NOVEMBER: 321(ST), 322(ST), 323(SM), 324(SM), 325(SM), 326(SM), 327(SM), 328(SM), 329(HW), 330(EM)[c], 331(EM)[c], 332(EM)[c], 333(EM)[e], 334(EM), 335(SA), 336(SA), 337(SA), 338(SA), 339(SA), 340(SA), 341(SA), 342(CO), 343(ST), 344(ST), 345(ST), 346(IR), 347(IR), 348(CO), 349(ST), 350(HW), 351(HW), 352(SA), 353(SU), 354(HY), 355(PO), 356(CO), 357(HW), 358(HY).

DECEMBER: 359(AT), 360(SM), 361(HY), 362(HY), 363(SM), 364(HY), 365(HY), 366(HY), 367(SU)[e], 368(WW)[e], 369(IR), 370(AT)[e], 371(SM)[e], 372(CO)[e], 373(ST)[e], 374(EM)[e], 375(EM), 376(EM), 377(SA)[e], 378(PO)[e].

VOLUME 80 (1954)

JANUARY: 379(SM)[e], 380(HY), 381(HY), 382(HY), 383(HY), 384(HY)[e], 385(SM), 386(SM), 387(EM), 388(SA), 389(SU)[e], 390(HY), 391(IR)[e], 392(SA), 393(SU), 394(AT), 395(SA)[e], 396(EM)[e], 397(ST)[e].

FEBRUARY: 398(IR)[f], 399(SA)[f], 400(CO)[f], 401(SM)[e], 402(AT)[f], 403(AT)[f], 404(IR)[f], 405(PO)[f], 406(AT)[f], 407(SU)[f], 408(SU)[f], 409(WW)[f], 410(AT)[f], 411(SA)[f], 412(PO)[f], 413(HY)[f].

MARCH: 414(WW)[f], 415(SU)[f], 416(SM)[f], 417(SM)[f], 418(AT)[f], 419(SA)[f], 420(SA)[f], 421(AT)[f], 422(SA)[f], 423(CP)[f], 424(AT)[f], 425(SM)[f], 426(IR)[f], 427(WW)[f].

APRIL: 428(HY)[e], 429(EM)[e], 430(ST), 431(HY), 432(HY), 433(HY), 434(ST).

MAY: 435(SM), 436(CP)[e], 437(HY)[e], 438(HY), 439(HY), 440(ST), 441(ST), 442(SA), 443(SA).

JUNE: 444(SM)[g], 445(SM)[g], 446(ST)[g], 447(ST)[g], 448(ST)[g], 449(ST)[g], 450(ST)[g], 451(ST)[g], 452(SA)[g], 453(SA)[g], 454(SA)[g], 455(SA)[g], 456(SM)[g].

---

a. Beginning with "Proceedings-Separate No. 200," published in July, 1953, the papers were printed by the photo-offset method.
b. Presented at the Miami Beach (Fla.) Convention of the Society in June, 1953.
c. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
d. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
e. Discussion of several papers, grouped by divisions.
f. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
g. Presented at the Atlantic City (N.J.) Convention in June, 1954.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1954

PRESIDENT

DANIEL VOIERS TERRELL

VICE-PRESIDENTS

Term expires October, 1954:
EDMUND FRIEDMAN
G. BROOKS EARNEST

Term expires October, 1955:
ENOCH R. NEEDLES
MASON G. LOCKWOOD

DIRECTORS

Term expires October, 1954:
WALTER D. BINGER
FRANK A. MARSTON
GEORGE W. McALPIN
JAMES A. HIGGS
I. C. STEELE
WARREN W. PARKS

Term expires October, 1955:
CHARLES B. MOLINEAUX
MERCEL J. SHELTON
A. A. K. BOOTH
CARL G. PAULSEN
LLOYD D. KNAPP
GLENN W. HOLCOMB
FRANCIS M. DAWSON

Term expires October, 1956:
WILLIAM S. LaLONDE, JR.
OLIVER W. HARTWELL
THOMAS C. SHEDD
SAMUEL B. MORRIS
ERNEST W. CARLTON
RAYMOND F. DAWSON

PAST-PRESIDENTS

Members of the Board

CARLTON S. PROCTOR
WALTER L. HUBER

TREASURER

CHARLES E. TROUT

EXECUTIVE SECRETARY

WILLIAM N. CAREY

ASSISTANT TREASURER

GEORGE W. BURPEE

ASSISTANT SECRETARY

E. L. CHANDLER

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

DEFOREST A. MATTESON, JR.
Editor of Technical Publications

PAUL A. PARISI
Assoc. Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

FRANK A. MARSTON, *Chairman*

I. C. STEELE
GLENN W. HOLCOMB
ERNEST W. CARLTON
OLIVER W. HARTWELL
SAMUEL B. MORRIS